CHAPTER 4

OUTLET FACILITIES FOR EMBANKMENT DAMS

Section I. Basic Considerations

4-1. Approach Channel. The purpose of the approach channel is to convey the water from the reservoir to the conduit intake structure. In some cases, the channel may function for diversion of the river during construction. The outlet channel design, unless extremely long, is usually dictated by the outlet works size and alignment. The alignment of the approach channel should take advantage of the area topography to decrease the channel excavation. Excessive curvature in the outlet channel near multiple gate intake structures should be avoided to help prevent unequal distribution of flow through the gate passages.

4-2. Conduits and Tunnels for Embankment Dams.

a. Alignment. The alignment and grade of conduits and tunnels are governed by diversion, evacuation, and operating requirements; tailwater elevation; topography; foundation conditions; and location of the dam and spillway. It is desirable to design conduits or tunnels that are as straight in alignment as practical, since a bend increases the hydraulic losses and creates unbalanced flow downstream from the bend. If it is necessary to change the direction of flow, the change should be accomplished with a long, easy, circular curve. The curved section should be located as far upstream from the exit portal as feasible in order to improve the flow conditions in the stilling basin. A model study should be made for questionable cases. Flow around bends causes dynamic and static reactions against the conduit or tunnel wall which should be considered in design, particularly for free-standing steel conduits within tunnels. Conduits and tunnels should have adequate slope for drainage; and when appreciable foundation settlement caused by embankment loading is anticipated, the vertical alignment should contain sufficient camber to compensate for the settlement.

b. Conduit Elevation. As with sluices for concrete gravity dams (see para 3-3a), the reservoir appurtenance requirements play an important part in determining the elevation of the flood control conduit. The inlets must be set low enough to drain the reservoir as required (ER 1110-2-50[d]) with due consideration of the conduit elevation relative to stilling basin design. A conduit at a low level may have better foundation conditions and higher discharge capacity for diversion and other low pool level operation; however, a longer conduit may be required and poor stilling basin action may result from high tailwater levels. Higher level conduits may have shorter length and the best potential for good stilling basin action and good flow conditions through the conduit for all discharges; but foundation conditions may require its location to be farther from the river channel, and a larger conduit may be needed for diversion or design capacity.

c. Shape. Flood control conduits for embankment dams are usually either cut-and-cover or tunnel construction. Although some cross-sectional shapes are superior to others from a hydraulic standpoint, structural and construction considerations usually establish the type of cross section. A circular cross section is the most efficient section for a tunnel flowing full. Horseshoe-shaped and rectangular sections provide large flow areas at low depths, which make them desirable for diversion purposes. The discharge capacity decreases sharply when the depth of flow in a rectangular conduit increases from nearly full to completely full flow, since the wetted perimeter is suddenly increased. The oblong shape has depressed pressure gradients at the exit portal compared with other shapes, when the outlet chute walls act somewhat like a draft tube (see para 5-2d(2)). Hydraulic characteristics of several shapes are shown in plate C-5.

d. Spacing. Where more than one conduit or tunnel is required, the spacing affects the stilling basin and intake design. Multiple cut-and-cover conduits should be spaced as close together as structural requirements permit in order to allow use of a single stilling basin and a minimum width intake structure. EM 1110-2-2901[k] discusses the spacing of multiple tunnels from the standpoint of geological and structural requirements. If the tunnels are designed with individual stilling basins, the spacing at the outlet portal must be sufficient to provide the necessary width of stilling basin for each outlet.

Section II. Intake and Gate Facilities

4-3. Intake Structures. The types of intake structures commonly used include gated tower, multilevel, uncontrolled two-way riser, and/or a combination of these. Intakes and control gates for embankment dams are discussed as integral structures, but if designed as separate structures, the principles of the hydraulic design are essentially the same. The hydraulic design of the intake structure should address the problems of (a) head loss, (b) boundary pressures, and (c) vortices in the approach.

a. Loss Coefficients. Loss coefficients for conduit intake structures with all gates operating range from 0.06 to 1.32 times the conduit velocity head. Available data from various geometries and gate operating schedules are summarized in plates C-32, C-33, and C-34. It is recommended that for discharge calculations, conservative values be selected from these plates in accordance with the planned intake geometry. Many of the coefficients given include allowance for trash struts or fender losses.

b. Boundary Pressures. Pressure gradients for intake structures should be developed to show local average pressure changes resulting from flow velocity changes. These gradients are helpful in evaluating pressure conditions in intakes, gate passages, and transitions. They should be examined in terms of the conduit back pressure for the entire operating range. This can be done by applying the energy equation (eq 2-3) to local changes in areas. Average pressures do not reflect pressure fluctuations due to turbulence, and cavitation potential should be evaluated according to the criteria discussed in paragraph 2-16.

c. Vortices. Vortices at intake structures can affect intake efficiency and create a safety hazard to the public. Although vortices are usually associated with high discharges and shallow intakes, they have been observed at intakes submerged as much as 60 to 100 ft (items 43, 95, 125, 131, and 138). Antivortex devices have been installed at intakes located at shallow depths. The intensity of the circulation phenomena set up around an intake is a function of the submergence of the intake, the discharge, and the intake and approach channel geometry. Gordon (item 43) has developed design guidance for preventing undesirable vortices (intensity such that they draw air and surface debris into the structure) at power plant intakes (plate C-35). Data for observed prototype vortices at Enid (item 131) and Denison (item 125) Dams have been included in this plate. It is recommended that Gordon's curve for unsymmetrical flow be used for design purposes. Reddy and Pickford (item 95) have analyzed vortex data pertinent to pump sumps and published a design chart for evaluating vortex potentiality for these structures. They concluded that when vortex prevention devices are used the critical submergence (ratio of water depth above top of inlet to inlet diameter - both dimensions at the entrance to the inlet bell mouth) should equal or exceed the inlet flow Froude number (otherwise, it should equal or exceed Froude number plus one) to provide vortex-free operation. Model studies are suggested in questionable instances.

d. Trashracks and Struts. If protection against clogging or debris damage to gates or turbines is needed, see the design guidance given in paragraph 3-5.

4-4. Intake Tower Versus Central Control Shaft. Both the intake tower and the central control shaft have their respective advantages. The intake tower may be expected to have higher back pressure at the gate section caused by the friction loss of the long downstream conduit. This is an advantage in the elimination of possible cavitation. As the intake tower has gates near the upstream end of the conduit or tunnel, the danger of leakage into or out of the embankment or abutment, with resultant piping of the material, is minimized. When the gates are placed near the upstream end of a conduit, there is the important advantage of being able to unwater the entire length of conduit for inspections. A central control shaft, which is usually located in an abutment near the axis of the dam, has the advantage of being protected from freezing and thawing and from forces due to ice action. In a central control the intermediate pier or piers are subject to high velocities and are designed to eliminate possible cavitation. The central control shaft has an advantage of not requiring a bridge for access as is the case of an intake tower. However, the conduit upstream of a central control shaft must be designed to withstand the reservoir head, and a transition is required both upstream and downstream of the gate passages. Foundation conditions and economic comparisons may dictate the choice between the intake tower and the central control shaft. Reservoir operating schedules may require the release of discharges under various heads and gate openings resulting in the pulsating flow condition ("burping") discussed in paragraph 2-4d. In some cases this undesirable condition can be eliminated by use of a central control shaft to shorten the conduit length downstream from the control gate. Further discussion of gate structure locations is given in EM 1110-2-2400.[j]

4-5. Submerged Intakes. The submerged intake is a comparatively simple and economical structure most often equipped with trash struts and bulkhead slots, having a streamlined entrance to the conduit or tunnel which is submerged at a low reservoir level. The submerged intake is satisfactory for reservoirs that function solely for flood control. However, when the intake will be permanently submerged by a conservation pool, difficulty arises in unwatering the conduit or tunnel upstream of the service gates. When bulkhead slots are located downstream from the intake face, provisions must be provided for closing the roof slot to prevent a high-velocity jet from entering through the slots and causing cavitation damages to the roof immediately downstream (item 20). Use of divers for bulkhead installation is to be avoided.

4-6. Combined Intake and Gate Structure. This is a common type of intake tower that usually requires a bridge for access, and gate wells are provided to accommodate the service and emergency gates. The emergency gate is upstream from the service gate and is utilized for inspection and maintenance of the service gate passage. The gate wells are generally wet for low head, wet-dry combination for intermediate head, and dry for high head structures. Determination of the well type is from structural and mechanical design considerations. A float well is normally provided for installation of a reservoir stage recorder. Bubbler gages are also used for this purpose and require less space. It is desirable to have two or three separate levels for the float well intakes, and they should be away from any drawdown effects when releasing large flows.

4-7. Underground Control Structures. An alternative to the conventional tower-type structure is an underground control structure buried beneath the embankment or in the abutment with a downstream access gallery. The access gallery should be placed adjacent to and at the same elevation as the water passages, essentially forming a multiple-box structure. Horizontal air vents will require check valves to prevent flow of water through them. The underground gate structures may be more economical than the conventional tower-dry well structure for high operating heads ($\geq$150 ft). Another economical advantage of this type of structure is the elimination of the service bridge which is required for a tower structure. Other conditions under which an underground structure should be considered include projects where water quality releases do not require multiple intakes over a wide range of reservoir levels and where reservoir operation results in periodic drawdown of pool level to the top of the intake bulkhead structure. Structural considerations are discussed in item 83. This type of structure has been used by others and by the Corps at the Fall Creek Dam in the Portland District and the New Hogan Dam in the Sacramento District.

4-8. Downstream Control Structures. Flow control facilities located at the downstream end of a conduit, when closed, subject the entire conduit to the full reservoir head and the possibility of high pressure leaks, piping along the conduit, and subsequent failure of the embankment. Therefore special design precautions are necessary when the control structure is located at the downstream end of the outlet conduit. The conduit between the impervious cutoff and the control structure may be a freestanding steel conduit housed in a concrete-lined tunnel of sufficient size to permit access for maintenance. This type of construction is frequently used for penstocks through embankments. Outlet facilities with downstream control must also have an emergency gate upstream of the steel conduit and stop log provisions at the conduit entrance. Provisions must be made for continued releases as required during shutdowns of primary release facilities.

4-9. Gate Passageway Requirements. The requirements discussed for sluice gates in paragraph 3-12 also apply to control gates for conduits through embankment dams. Normally a service gate, an emergency gate, and slots for bulkheads or stop logs should be provided for each gate passage to the conduit or tunnel. The total flow cross-sectional area of gate passages should exceed the downstream conduit area by 10 to 15 percent. Typical gate installations for both tainter and vertical-lift gates are shown in plate C-36.

4-10. Low-Flow Releases. The operation of large gates at small openings (<0.5 ft) is not recommended because of the increased potential for cavitation downstream from the gate slot. In cases where low-flow releases are required, consideration should be given to low-flow bypass culverts, center pier culverts, multilevel wet well facilities (see Chapter 6), or a low-flow ("piggy-back") gate incorporated in the service gate.

Section III. Entrance Shapes

4-11. General. The general design of entrance shapes, discussed in paragraph 3-6, is equally applicable to conduits for embankments and concrete dams although the structural setting and some details are different. Entrances in concrete dams are ordinarily constructed as bell mouths for circular conduits and with entrance curves at the top, bottom, and sides for rectangular conduits. In embankment dams, the conduit inverts are normally set at approximately the same elevation as the floor of the approach channel. Consequently, there is little curvature of the invert approach so that a bottom curve is not required. In the case of embankment dam intakes with two or more gate passages, there usually is insufficient lateral space for full bell-mouthed entrance curves on the sides, so that only the roof is bell-mouthed and the piers and sides are extended upstream to support the trash struts. The sides and piers are carefully transitioned from rounded noses to the gate passage. In the case of a single rectangular gate passage, the top and sides can be flared or treated as above.

4-12. Selection of Entrance Shape for Design. A comprehensive series of tests on flared entrances has been conducted at WES (item 76). Intake roof curves for conduits with fully suppressed intake inverts and limited lateral space for side flares should be designed as indicated in plate C-37. The short elliptical shape (fig. a, plate C-37) is satisfactory when the back pressure on the intake is great enough to prevent low local pressures. The long elliptical shape should be used when back pressure is not adequate to eliminate low local pressures (fig. b, plate C-37). The effects of upstream face geometry are given in HDC 221-2[n] and item 20. Intakes with sufficient lateral space for sidewall streamlining should have curves as shown in plate C-22 and discussed in paragraph 3-6c.

4-13. Linear Sidewall or Pier Flare. WES studies show that entrance roof pressure conditions for two-dimensional curves can be improved by tapering the divider piers. Plate C-38 shows the improvement of pressure conditions from using linear sidewall and/or pier flare. The computational procedure is illustrated in HDC 221-3 and 221-3/1.[n] Two-dimensional roof curve pressure coefficients can be converted to three-dimensional coefficients for side flare by:

$$C_3 = C_2\left(\frac{A_2}{A_3}\right)^2 \qquad (4\text{-}1)$$

where

C = pressure drop coefficient

A = flow area in square feet at the point of interest

Subscripts 2 and 3 indicate two- and three-dimensional values, respectively. Unless model-tested, it is recommended that application of equation 4-1 be limited to the cases where the horizontal flare does not exceed 1 horizontally to about 12 longitudinally.

Section IV. Control Gates

4-14. General. The types of gates and valves and their operating characteristics discussed in paragraphs 3-8 to 3-17, are equally applicable to conduits for embankment dams. Generally, a service gate, an emergency gate, and slots for bulkheads or stop logs are provided for each gate passage to the conduit or tunnel (plate C-36). Cable-suspended tractor or hydraulically operated tractor or slide gates are normally used in conduits for embankment dams. The problems of determining the hydraulic forces acting on tractor gates, with emphasis on cable suspension, will be discussed in this section. Although downpull forces on a partially opened gate constitute a hoist design problem in both hydraulically operated and cable-suspended gates, the vibration problem is more critical in the design of cable-suspended gates. For this reason cable-suspended tractor gates are not recommended for flow regulation or for heads in excess of 150 ft.

4-15. Gate Lip Geometry. Laboratory and field tests have shown that the 45-deg gate lip design shown in plate C-23 performs satisfactorily under all flow conditions. The 45-deg lip should terminate in a 1-in.

vertical extension to ensure that the jet springs free from the upstream edge of the lip. The upstream face should be flat rather than "bellied" in order to have uniform flow conditions across the width of the conduit.

4-16. Vertical-Lift Gate Discharge Computations. Plate C-39 presents a suggested design discharge coefficient curve for use with equation 3-2 for developing rating curves for vertical-lift gates with 45-deg bottoms (plate C-23) and assuming free-surface flow downstream of the gate. Stage-discharge relations for selected gate openings and for free-surface flow downstream can be computed with CORPS H3201.[o] Single gate passage structures of nominal length and reservoir head generally have downstream free-surface flow for gate openings up to 80 percent of the gate passage height. In multiple gate passage structures, this 80 percent value may be greatly reduced with two or more gates partially open. In any event, computation of the flow profile between the gate and conduit exit portal is necessary to ascertain the gate opening at which flow control shifts from the gate to the exit portal due to conduit friction for a given pool elevation, thus possibly causing flow pulsations ("burping") as discussed in paragraph 2-4d. Generally, the downstream conduit slope is mild and the flow profile will be the M_3 type (see plate C-1). Therefore, an initial depth in the downstream conduit proper must be estimated and the profile computation proceeds in the downstream direction. For a single passage structure, or for a multiple passage structure with balanced operation, it is recommended that this depth be estimated from the jet vena contracta and assume an energy loss between the gate and the conduit proper (transition loss) of 0.1 times the jet velocity head. For unbalanced gate operation, it is recommended to assume this energy loss at 0.2 times the average jet velocity head.

4-17. Commercial Gates. There are many commercially available slide gates, tainter gates, knife-gate valves, flap gates, etc., that are readily adaptable to low head and small discharge flood control and drainage projects. HDC 340-1[n] presents head loss coefficients for flap gates used extensively in flood protection and drainage projects. Pickett et al. (item 93) have compiled considerable data on discharge and head loss coefficients for various types of gates and valves.

4-18. Hydraulic Load for Vertical-Lift Gates.

a. General. The hydraulic load on the gate leaf should be determined both for gate closed and part gate operation. Hydraulic loads are computed in the usual manner with the gate closed and the reservoir at maximum level. The vertical hydraulic loads on the gate during partly open operation can be separated into upthrust on the bottom and downthrust on the top as indicated in HDC 320-2.[n] The upthrust for 45-deg gate bottoms determined from model and prototype tests is shown in HDC 320-2/1.[n] Both slide gates and tractor gates are included. The unit upthrust load is in terms of the effective head on the gate. A similar type of graph for downthrust on the top of the gate is shown in HDC 320-2/2.[n] The data on downthrust are applicable only to gates with similar upstream and downstream clearances between the gate and the roof slot boundaries. HDC 320-2/3[n] presents a sample computation illustrating the use of HDC 320-2/1 and 320-2/2[n] in the solution of a hydraulic and gravity force problem. Additional hydraulic load data have been reported by Simmons, Naudascher et al., and Smith (items 113, 79, and 115, respectively). The occurrence of free-surface or full conduit flow downstream from the gate, the transition from either to the other, and the gate speed may have considerable effect on the hydraulic load.

b. Gate Catapulting. An intake gate is sometimes used for rapidly watering-up the penstock and turbine scroll case, or the space between the service and emergency gates, by simply opening the intake gate a few inches. As the space between the intake gate and downstream gate becomes full, the water may rise through an opening between the downstream side of the intake gate and the gate slot. If this back-of-gate opening area is smaller than the gate opening area, it may restrict the vertical flow of water into the gate slot. Under these conditions sufficient hydraulic forces on the gate have occurred at several projects that would abruptly raise or "catapult" the gate tens or even hundreds of feet up the slot (items 40 and 98).

4-19. Vibration of Cable-Suspended Gates. Thompson (item 121) treats the theory of vibration with the determination of whether any disturbing frequencies are inherent in the hydraulic system of a design that may approach the natural frequency of elements of the system (gates, valves, splitter piers, stilling basin walls, etc.). As the magnitudes and frequencies of the exciting hydraulic forces can only be approximated in most cases, it is necessary to effect conservative designs. Fortunately most of the exciting hydraulic forces have high frequencies and the natural frequencies of the various elements of the structure are very low. The case of an elastically suspended conduit gate is used to illustrate application of the theory.

a. Resonance. When the forcing frequency is exactly equal to the natural frequency a condition of resonance exists. The displacement amplitude for the vibrating system increases without bound and is governed only by the amount of damping in the system. This may result in structural rupture. The amplitude can also increase rapidly if there is only a small difference between the forcing and natural frequencies. The undamped magnification factor

$$\frac{x}{x_o} = \frac{1}{1 - \left(\frac{f_f}{f_n}\right)^2} \tag{4-2}$$

where f_f/f_n is the ratio of the forcing frequency to natural frequency, represents the factor by which the zero frequency deflection x_o of the spring-mass system under the action of a steady force must be multiplied to determine the amplitude x. It is desirable to produce a design with a low magnification factor.

b. Forcing Frequencies. Two possible sources of disturbing frequencies are the vortex trail shed from the bottom edge of a partially open gate and the pressure waves that travel upstream to the reservoir and are reflected back to the gate. The frequency of the vortex trail shed from a flat plate oriented with face perpendicular to flow direction can be defined by the dimensionless Strouhal number, S_t, as follows:

$$S_t = \frac{L_p f_f}{V} \tag{4-3}$$

where

L_p = plate width

f_f = vortex trail shedding frequency

V = velocity of the fluid

The Strouhal number for a flat plate is approximately 1/7. The forcing frequency of a vortex trail shed from a gate may be estimated as:

$$f_f = \frac{\sqrt{2gH_e}}{7(2Y)} \tag{4-4}$$

where

H_e = energy head at the bottom of the gate

g = acceleration due to gravity

Y = projection of the gate into the conduit or half of the plate width L_p

Unpublished observations of hydraulic models of gates have indicated that the vortex trail will spring from the upstream edge of a flat-bottom gate causing pressure pulsations on the bottom of the gate. The vortex trail springs from the downstream edge of a standard 45-deg gate lip, eliminating bottom pulsations. A more recent research study at Iowa (item 60) on flat-bottom gates indicates that the 45-deg sloping gate bottom used by the Corps should be free of vibration induced by vortices shed from the gate lip. The frequency of a reflected positive pressure wave may be determined from the equation:

$$f_f = \frac{C}{4L} \tag{4-5}$$

where

C = velocity of the pressure wave

L = length of the conduit upstream from the gate

The pressure wave velocity is dependent upon the dimensions and elastic characteristics of the pipe or of the lining and surrounding rock of a tunnel. Data are given in HDC 060-1/2[n] by Parmakian (item 90) for various combinations of these variables.

c. Natural Frequency. The natural frequency of free vertical oscillation of a cable-suspended gate can be expressed by the equation:

$$f_n = \frac{1}{2\pi}\sqrt{\frac{gE}{12\ell\sigma}} \tag{4-6}$$

where

E = modulus of elasticity of the cable

ℓ = length of the supporting cable

σ = unit stress in the cable

d. Sample Computation. HDC 060-1/4 and 060-1/5^n present sample computations illustrating the above theory.

Section V. Transitions

4-20. General. Transitions are required to effect changes in conduit size (expansions and contractions) and shape (rectangular to circular, circular to rectangular, etc.). They may be abrupt with high head loss or streamlined with small head loss depending upon the purpose. At Mica Dam abrupt expansions have been designed as in-line energy dissipators (item 104). Singh (item 114) has recently presented a procedure for designing a streamlined circular-to-rectangular transition resulting in essentially a straight-line variation in area effecting improved hydraulic performance. Transitions fall into three general categories: (1) entrance, (2) in-line, and (3) exit. In flood control conduits, transitions are used to connect a usually rectangular gate passage to circular- horseshoe- or oblong-shaped conduits. They are also used at conduit exits to help spread the flow prior to entering the energy dissipator. In sluices they are frequently used to effect exit portal constrictions, to increase sluice back pressure, and to spread the jet on the spillway face.

4-21. Entrance and Intake Transitions. Entrance transition design for both circular and noncircular inlets has been discussed in paragraph 3-6. Typical entrance transitions are shown in plates C-21, C-32, and C-33. From the data presented in these plates, it can be assumed that loss coefficients for well-designed simple entrances will not exceed one-tenth the conduit velocity head. In complex intakes, the entrance loss is included in the combined intake loss. Comparable entrance pressure data are given in plates C-22, C-37, and C-38.

4-22. In-Line Transitions.

a. Location. Water usually flows through several different passageways in its route from the reservoir to the river below the dam. Transitions have the function of providing a smooth change from one cross section to another in such a manner than hydraulic losses and cavitation potential are minimized. Transitions are generally required at one or more of the following locations: (1) between the intake gate passage and the upstream end of a circular conduit, (2) upstream and downstream from a central control gate passage, and (3) at the outlet end of the conduit. If gate passages are the same height as the downstream conduit, double curvature of the transition fillets will be avoided.

b. Smoothness in Direction of Flow. A well-designed transition should provide a gradual change in area boundary shape. The transition boundaries should follow easy curves, with intervening tangents if required, and the curves should be well defined to facilitate construction. The maximum change in flow direction occurs along the path of convergence of the outside corners of the transition. The junction of the corners of the transition with the desired section downstream should be carefully checked to determine whether the desired curvature is obtained to prevent the occurrence of separation or negative pressures along corner boundary lines. Construction joints should not be located at or near the end of the transition. Since a negative direction change of boundary (away from the flow direction) reduces pressure, any misalignment of construction forms or subsequent small movement of monoliths on either side of a joint may further accentuate the drop in pressure and cause cavitation. (See item 7.)

c. Length. The required length of the transition as compared with the conduit diameter depends upon the lateral, vertical, and diagonal boundary changes. The number and arrangement of gate passages also affect the length of the transition. As the number of gate passages increases, the length of the transition generally increases. As a general rule, to eliminate the possibility of cavitation damage within and just downstream of the transition and to minimize head loss, the ratio of a contraction transition length to maximum radial offset from the outside boundary of the gate passage to the corresponding location on the conduit boundary should be about $V/\sqrt{gD}$ (V and D being the average of the maximum average velocities and equivalent diameters at the beginning and end of the transition). However, for certain combinations of gates and tunnel sizes, this guideline may result in too severe contraction for low heads, in which case the length should be increased to reduce the angular rate of change along the transition. Thus, maximum angle of contraction or expansion relative to the conduit center line should be limited to about 7 deg. A sample computation for the design of transitions is presented in Appendix E. The procedure is applicable to all in-line transitions.

d. Pressure Gradients. A study should be made of the local average pressure throughout the transition. Average pressures can be computed using the Bernoulli equation (eq 2-3) and the average pressure should be equal to or greater than atmospheric pressure. Pressure data in transitions may be found in items 68 and 132 for entrance and midtunnel transitions, respectively.

4-23. Exit Transition. Normally the shape (circular, horseshoe, oblong, etc.) of an outlet conduit or tunnel for embankment dams is maintained to the exit portal and the transition into the energy dissipator is made in an open channel downstream from the portal. When the embankment slope is relatively flat, the tunnel or conduit can be shortened by moving the transition upstream into the embankment and abruptly raising the roof to ensure free-surface flow in the transition. Design details of a typical outlet works exit transition are presented in Chapter 5.